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PERIODIC NONDESTRUCTIVE EVALUATION
OF IN-SERVICE METAL BRIDGES

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16. Abstract Metal bridges are subject to in-service fracture problems mainly caused by fatigue. This report considers the prevention of bridge failure by the performance of periodic nondestructive evaluations. Differences in visual and other nondestructive inspections are discussed. The use of preliminary inspection strategies for proper testing also is presented. The suitability of common types of nondestructive testing for bridge inspections also are included.			
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INTRODUCTION

Metal bridges are subject to in-service cracking that may cause disabilitation or failure. These insidious attacks have been the subject of many previous studies. The continual occurrence of cracking on metal bridges nationwide indicates that further efforts are needed to preclude these events. The relative infrequency of major cracking problems is more than offset by the great costs encountered when afflicted bridges fail or require repairs.

During the past 35 years, many typical metal bridges have suffered major cracking problems. Those bridges include the Duplessis Bridge, Quebec, Canada (1950); the Kings Bridge, Melbourne, Australia (1962); the Silver Bridge, Point Pleasant, West Virginia (1967); the Bryte Bend Bridge, Sacramento, California (1970); the Fremont Bridge, Portland, Oregon (1971); the Quinnipac Bridge, New Haven, Connecticut (1973); the I-24 Bridge, Paducah, Kentucky (1975); the I-79 Bridge, Neville Island, Pennsylvania (1978); the US Grant Bridge, Portsmouth, Ohio (1978); and the US-18 Bridge, Prairie DuChien, Wisconsin (1981).

Some of the problems may be related to environmentally assisted corrosion processes. However, most cracking problems in metal bridges may be related to the welding process in fabrication and to the cyclic loading (fatigue) in service. Welding significantly increases the chances of introducing subcritical or critical-size defects in a structure during fabrication. Fatiguing loads are practically unavoidable in many cases. Those loads allow subcritical crack growth at service-level stresses. When a crack reaches a critical size in a tensile or flexural loading situation, the afflicted structural member will usually fail catastrophically.

The interaction between welding-induced defects and fatigue is extremely damaging. Welding defects may grow rapidly into critically sized cracks in a cyclic-loading environment. Some welding defects, such as slag inclusions or porosity that would not be harmful in a static-loading environment, become potential sources for fracture when subjected to cyclic loading.

Welding defects also may contribute to fracture problems in a static-loading environment. Improper welding procedures together with poor inspection may admit critical-sized cracks into structures. Welding processes also may introduce residual or reaction stresses that act as driving forces for catastrophic fast fracture. Combined with low temperatures, these factors have resulted in the brittle fracture of several bridge members.

Shop inspection of welded bridge members is now commonplace. Many states have their own personnel perform nondestructive shop inspections. Although the conventional forms of nondestructive shop testing have been subject to recent criticism, those methods are widely accepted and technology is firmly in place.

The opposite is true for routine nondestructive testing of existing bridges. Although factors dictate the execution of such work, it is rarely accomplished. No form of nondestructive testing is widely recognized as effective for field inspections. The bulk of such work, when performed, is done by private testing companies. That work is very expensive and results are sometimes unsatisfactory. The need exists to

incorporate existing techniques and develop new methodologies to allow state highway agencies to perform periodic nondestructive field inspections of bridges in an economical and effective manner.

This paper discusses some technical aspects pertinent to the planning and performance of periodic nondestructive evaluation (NDE) of in-service bridges. Work relevant to this topic is presently being conducted by the University of Kentucky Transportation Research Program for the Kentucky Transportation Cabinet in a Federal Highway Administration participating study KYHPR-84-95, "Special Problems of Metal Bridges."

TYPES OF INSPECTIONS

Presently, bridge inspections in Kentucky, as in most states, fall into two extremes of the NDE scale (Figure 1). Recent experience has shown obvious dangers inherent in the complete lack of inspection or "Trust Fate" attitude that results in the lowest short-term cost for the bridge owner but also entails the highest risk.

Perhaps an example of the dangers of this approach to bridge inspection was the Kings Bridge in Melbourne, Australia, which failed in July 1962 under the load of a heavy truck. Apparently, a deck girder had fractured during the first winter of service. That event went undetected until the bridge collapsed (1, 2). The bridge had a four-girder deck system, which was probably load-redundant. However, the loss of one girder caused the other girders to fail catastrophically in a relatively short period of time. That bridge collapse indicated that even redundant bridges may require inspection of a higher order.

Federal law requires that all bridge structures located on federal routes be inspected at least once every two years by a professional engineer or by personnel having completed specialized training in maintenance inspection of bridges. Sometimes, those inspections may be too superficial to detect cracks that could affect the structural reliability of bridges. Those inspections are "walk-overs" by a few personnel who also must be concerned with non-related matters such as the function of bridge lights, the condition of paint, and the quality of the bridge deck.

A typical biannual inspection of a large Ohio River bridge may be performed in one-half day with a crew of four to six. Most bridges have limited access to critical structural areas thereby preventing or restricting visual crack detection. Usually, inspection of girders or tie-chords must be accomplished from central catwalks under the bridge deck, if such access is even provided.

One useful means of cursory examination is the suspension bridge cable inspection port (Figure 2). Those devices have hatches that may be readily removed to allow inspection of the lower cable strands for wire corrosion damage (Figure 3). A few of those devices placed on low points of bridge cables can provide a fairly representative inspection of the interior strands that are covered by wrapping over the remainder of the cables.

An intermediate form of inspection, superior to the federally mandated biannual inspections, is the comprehensive visual inspection. However, there are several unfavorable aspects to this type of

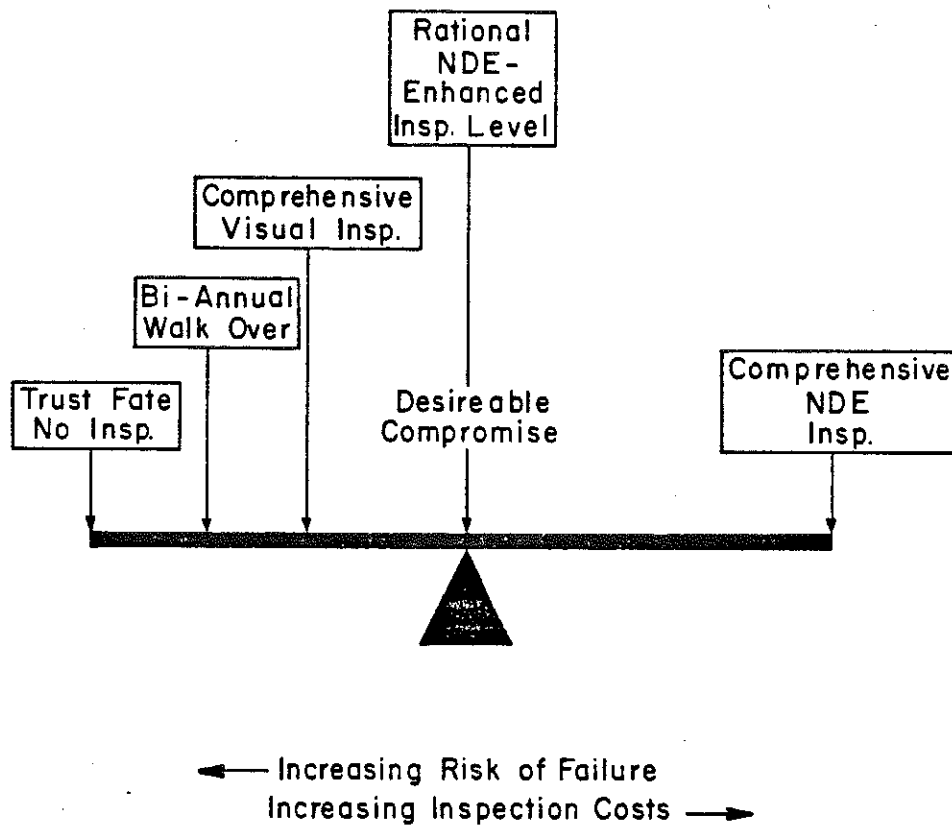


Figure 1. Scale of Bridge Inspections.



Figure 2. Suspension Bridge Inspection Port.

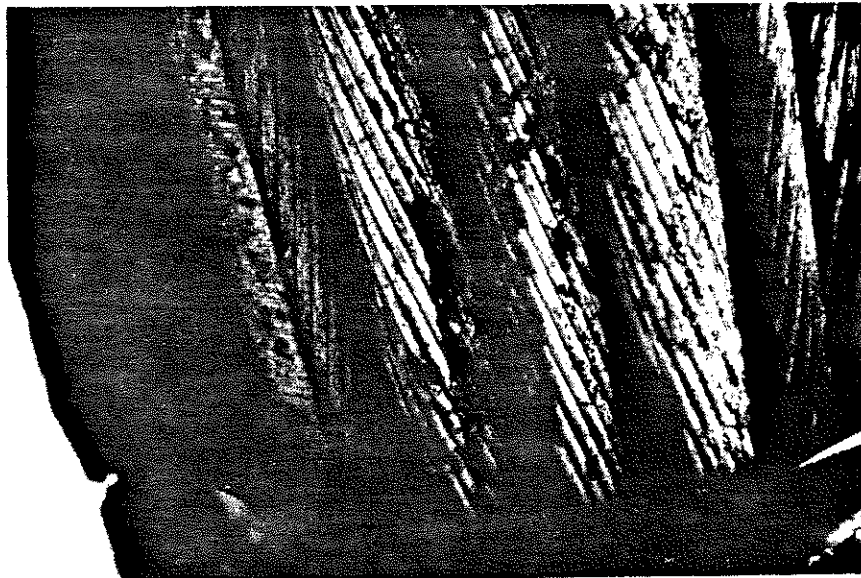


Figure 3. Interior View of Wire Strands.

inspection, especially when compared to NDE-enhanced inspection techniques. Visual inspection is limited to surface-breaking flaws. Visual inspectors must use the same equipment (snoopers and lift buckets) to access structural members as an NDE operator. Those inspectors must have some physical and technical qualifications. Proper comprehensive visual inspection of bridge elements may involve paint removal and subsequent repainting. A NDE method should be employed to verify any indication detected by visual inspection. Also, total visual inspection costs may exceed some testing costs involving NDE methods.

Evidence exists indicating that comprehensive visual examination may not provide sufficient assurance of structural integrity. In 1950, the Duplessis Bridge at Quebec, Canada, was closed for a period of 10 days for visual inspection of the bridge for cracks in the deck girders (which previously had been repaired). No defects were found and the bridge was reopened to traffic. Two weeks later, the west portion of the bridge collapsed (3).

This should not be taken to imply that visual inspection is entirely unsatisfactory. Many cracks in bridges are sufficiently large and have been visually enhanced by corrosion as to allow easy detection by relatively unskilled observers. Many fatigue cracks on Kentucky-owned bridges have been detected by painters. However, it is an unsound practice to place verification of structural integrity in the hands of persons whose primary function and training is not in crack detection. Also, it may be unwise to correlate the inspection frequency of some bridges with that of painting operations.

On the high-cost end of the inspection scale envisioned in Figure 1 is the comprehensive nondestructive inspection. Historically, that type of work has been performed on bridges for three reasons: either a crack was previously observed on the subject bridge, the extent or accuracy of the fabrication quality assurance was questionable, or similar bridges had experienced cracking problems. Poor fabrication quality-assurance record keeping could be a contributing factor in each case.

Usually, comprehensive nondestructive inspection entails the use of one or more NDE consultants who perform inspections through use of a number of conventional NDE methods such as ultrasound, radiography, magnetic particle, or dye penetrant (Figure 4). Subsurface defects detected by ultrasound or radiography often are removed by coring and taken to a laboratory for examination by sectioning or tomography (4, 5).

Unfortunately, while this approach may detect cracks that exist on inspected members of the bridge, it also has some drawbacks. The cost of NDE testing is very expensive and may approach \$250,000 for a large bridge. As will be discussed, such inspections may raise as many questions about the presence of potential defects as they answer. Testing may lead to traffic disruptions lasting for several months. When test results indicate no defects, even if only a small percentage of the bridge's fracture-critical members are inspected, bridge authorities may conclude that the structure contains no potential or undetected defects. The structure may never again be closely inspected.

In some instances, comprehensive nondestructive inspections of bridges may be warranted. A bridge might exhibit cracking in its fracture-critical members. Another case might be bridges whose key structural members are difficult to access. Occasionally, bridges have

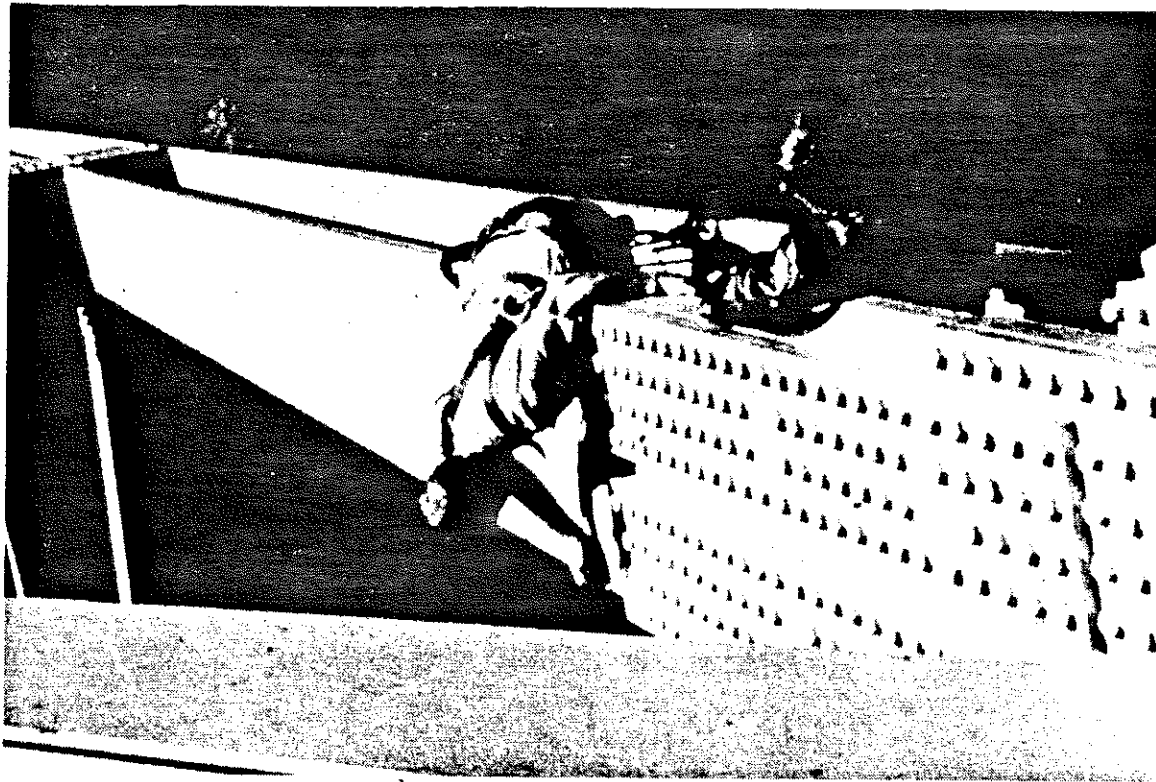


Figure 4. Consultant Inspecting a Bridge Using Ultrasonic Testing.

key structural members subject to negligible cyclic loading. Those members may not incur fatigue cracks. But, early in the service life of those bridges, it may be desirable to conduct a single inspection for critical-sized cracks. Usually, however, limited NDE funds available are better spent protecting the public by employing other approaches to bridge nondestructive inspection such as:

- (1) Expending allocated funds on less extensive inspections of several bridges and

- (2) Conducting less extensive inspections of a bridge, but repeating the inspections at more frequent intervals.

Problems with comprehensive NDE inspections may occur when the tests are performed to fabrication codes such as the American Welding Society (AWS) "Structural Welding Code -- D1.1." This is especially true of ultrasonic inspections of subsurface defects in welds of older bridges that were not fabricated to provide for ultrasonic testing.

Between 1979 and 1982, Kentucky Department of Highways personnel conducted extensive ultrasonic tests on butt-welds of a large tied-arch bridge (Figure 5). At some 40 different locations, ultrasonic indications of subsurface flaws were detected; several of those indications exceeded permissible AWS limits for bridges. One of those locations was cored and the coupon was taken to a laboratory where it was sectioned (Figure 6). The resulting core revealed a harmless lamination in the base metal adjacent to the weld (Figure 7). Such laminations also have caused ultrasonic defect indications in another field test (5). Other suspect locations on that bridge containing AWS Code defect indications were radiographed, but no defects were revealed.

Typically, this leaves the inspector in a dilemma. Should he neglect the ultrasonic results, which are usually more crack-sensitive than radiographic tests? For greater assurance, the bridge owner may core those locations. However, that work is expensive and yields a somewhat weakened structure.

Considering that only limited NDE funds may ever be available to bridge authorities, it is desirable that some compromise be achieved between nominal inspection of all bridges within the jurisdiction and heavy financial expenditures on a single bridge. This does not necessarily mean the extent of testing on a bridge needs to be reduced in terms of items inspected. Rather, simple and more economical NDE procedures should be employed.

The type and size of defect to be detected does not have to be closely related to the codes or specifications to which the structure was constructed. Consideration of rejectable flaws may be limited to cracks of given minimum size and disposition. As larger sizes of maximum permissible flaws are sought, they become easier to detect by NDE. Also, the inspection time may be reduced significantly, thereby reducing inspection costs for a structure.

When defects (cracks) are detected by such inspections, more comprehensive nondestructive inspection of a bridge may be performed with justification. However, under most circumstances, inspections of bridges should not be considered final or "one-shot" affairs. There are two main reasons for this. First, flaws may be overlooked even by conscientious, competent inspectors. Second, subcritical fatigue crack growth may occur with time, and in several years the structural integrity of a bridge can be threatened by growing cracks. Proper NDE

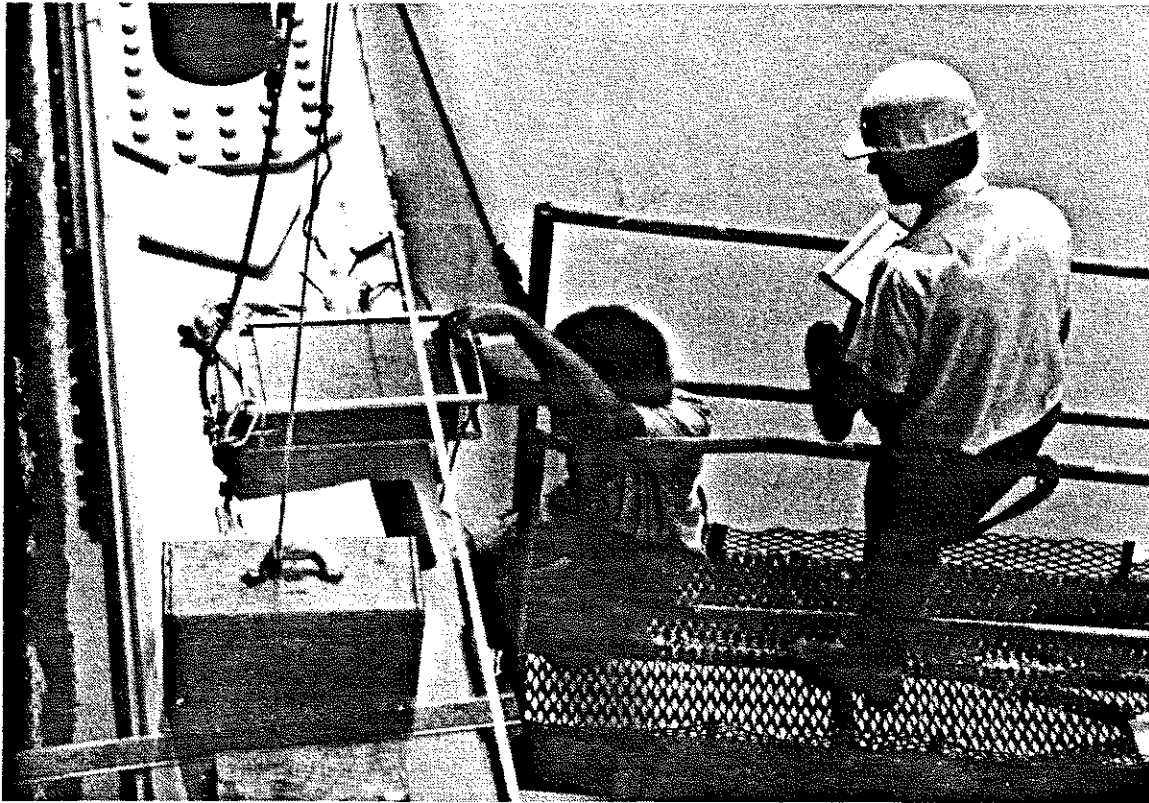


Figure 5. Ultrasonic Inspection of a Butt Weld on a Tie Chord of the I-471 Bridge.

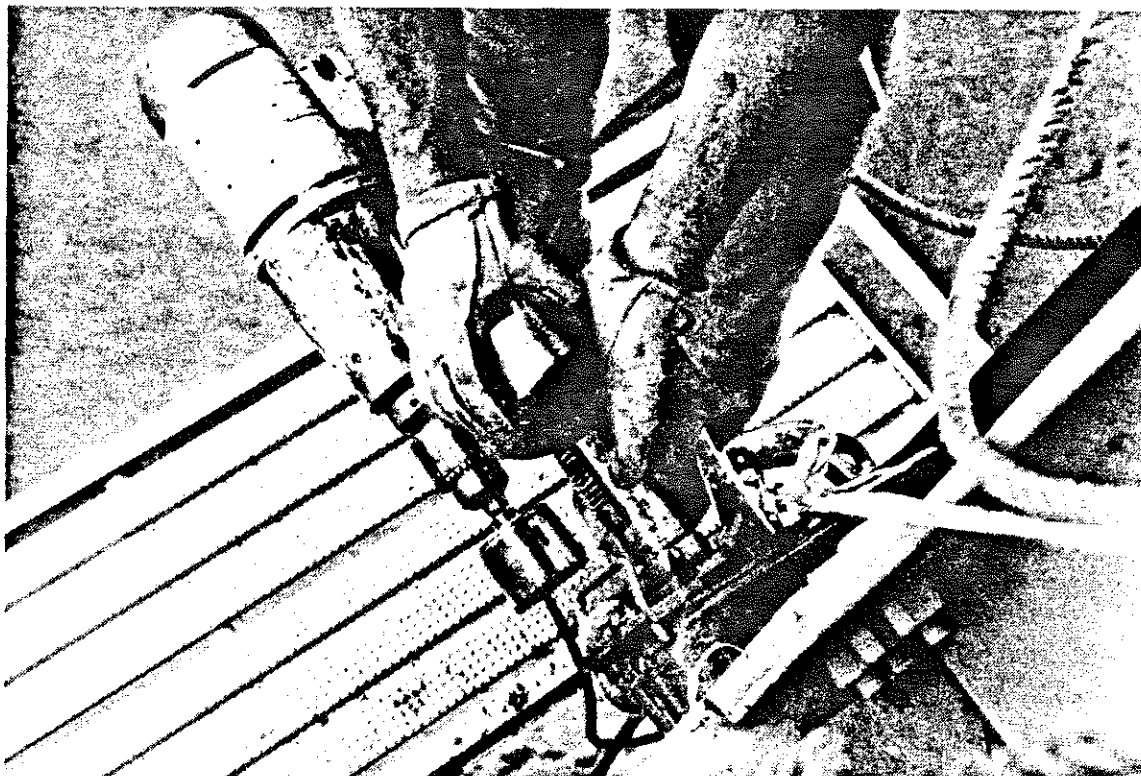


Figure 6. Core Taken from a Butt Weld on the I-471 Bridge.

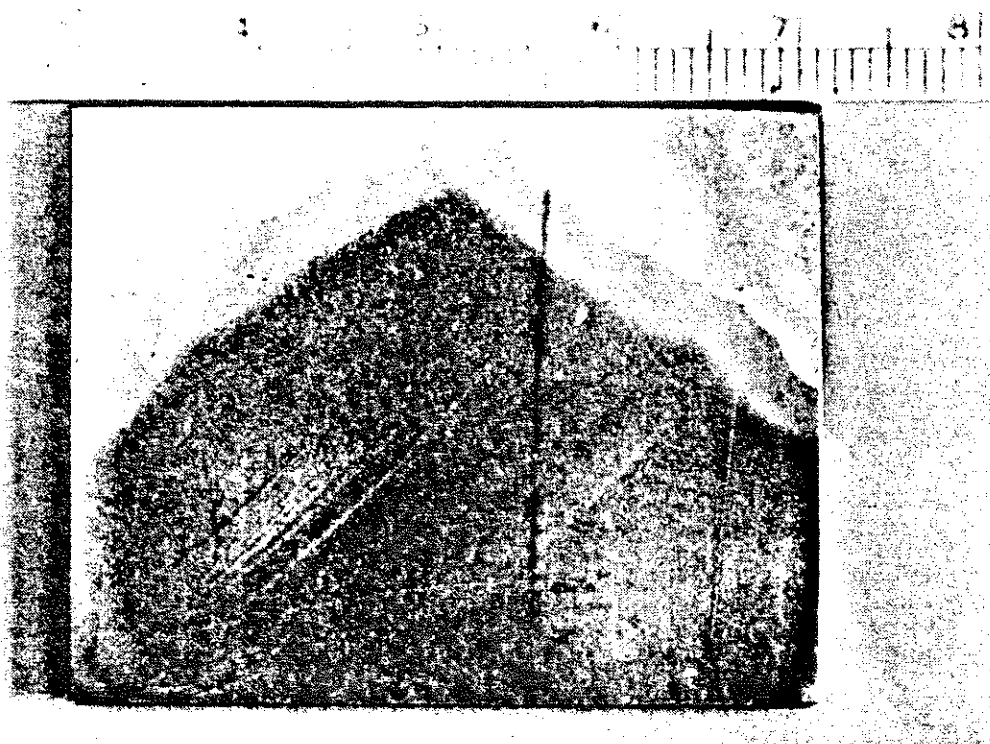


Figure 7. Lamination Found in a Sectioned Core.

scanning, conducted at reasonable intervals, will be able to detect growing cracks before they damage or destroy a bridge.

INSPECTION STRATEGIES

Inspection or reliability strategies are written plans set forth by bridge authorities as rationale for impending inspections. The formulation of those plans is necessary to ensure that any efforts expended will produce desired results (e.g., assurance of structural integrity of the bridges inspected).

Inspection strategies should be prepared prior to the performance of actual field inspections. They may be employed to 1) define the purpose and scope of NDE tests, 2) aid in requesting funds, 3) select candidate bridges, 4) determine inspection frequency, and 5) choose appropriate test method(s).

Due to differing circumstances, strategies employed by each state may vary. The rationale and focus of the strategies also may differ. Therefore, inspection strategies may contain a wide variety of information including historical data, estimated failure costs, estimated risks, bridge inventories, estimated inspection costs, traffic data, bridge design loadings and criteria, weather data, fracture mechanics data, reliability assessments, previous inspection reports, inspector requirements, and equipment requirements. Also, many reliability and risk assessment techniques have been formulated by structural, energy, aircraft, and naval researchers (6-11). Those may be adapted for use as bridge inspection strategies. A few items related to bridge inspection strategies will be briefly discussed.

In preparing inspection strategies for bridges, both structural risk and human risk may be considered. These items usually are interdependent and may be combined to provide an accurate indication of not only the total risk but also the anticipated consequences of bridge failure.

Structural risk depends on: 1) structural redundancy, 2) loading history, 3) present loading, 4) anticipated future loading, 5) structural details (i.e., AASHTO fatigue categories), and 6) bridge environs (e.g., atmosphere, approaches, highway geometrics, and bridge deck profile).

Historical data suggest that, since the turn of the century, a major bridge in the United States has collapsed or failed structurally about once every 15-20 years. Based on simple probability, the odds against bridge failure for a state in a given year are about 1,000 to 1. While those odds at first glance seem to preclude failure, combined with other data, they may be used as a crude justification for funding.

Figure 8 shows a failure rate versus time (bath tub) curve that is typical for a multitude of manufactured items ranging from electronic components to bridges (12). The initial or "burn-in" portion of the curve shows a higher failure rate than the middle portion of the curve. Bridge failures that occur in this portion of the curve are usually caused by poor construction materials, improper weld techniques and repairs, and fabrication defects missed by quality assurance inspections. Many recent bridge problems due to weld cracking may be considered "burn-in" failures. In the middle portion of the curve or

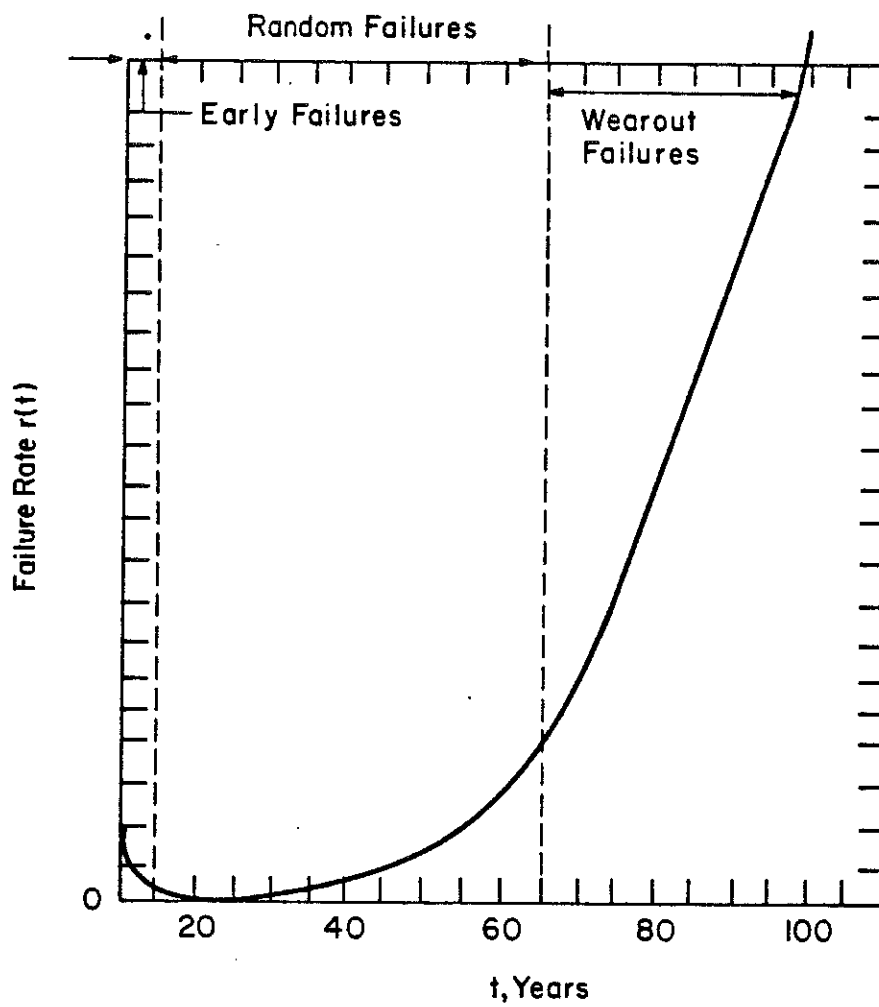


Figure 8. Failure Rates - vs- Time (Bathtub Curve)
(Ref. II, pp 167).

"prime-of-life," failures occur randomly in an unexpected manner (termed catastrophes). An example of this was the Silver Bridge failure at Point Pleasant, West Virginia, in 1967.

The "wear-out" or "burn-out" final portion of the curve reflects the cumulative effects of corrosion and subcritical crack growth. Those events also are termed "on-line" failures as they should be anticipated. Any bridges that exceed their original design or anticipated service lives may be subject to "on-line" failure.

Human risk due to structural collapse will differ between bridges due to many factors: 1) average number of motorists on the bridge at any time, 2) maximum number of motorists on the bridge at specific times, 3) physical consequences of collapse (fall distance, covering debris, and underlying water), and 4) highway geometrics. As shown in Table 1, existing generalized human risk data may not, at a glance, support the need for periodic NDE surveillance for many types of bridges. However, one must assume that bridge failure constitutes an involuntary risk, whereas driving usually entails voluntary risk. Involuntary risks should be 3-4 times less than voluntary risks to be considered equivalent based on present social values. When bridge collapse risk exceeds the normal risk exposure for motorists, inspections are easily warranted.

Even more justification for periodic nondestructive inspections may be based upon consideration of the total consequences of bridge collapse or structural dilapidation. Major direct costs of bridge failure may include: 1) litigation due to loss of life or injury, 2) structure replacement or repair, 3) provision for alternate traffic routing, 4) accident investigation, and 5) clearing of underlying waterways. It is difficult to determine the total cost of these factors.

Several other indirect consequences must be taken into account when such major problems occur. Key personnel may consume a major portion of their time attending to a bridge failure. Much of a year's planning and construction budget for a highway department may well be consumed in coping with the event. Also, upper-level management may be occupied by major and nuisance litigation for several years. Political expediency also may result in the unwarranted dismissal of some vital personnel. Communities and businesses that depend on the bridge for their economic welfare may suffer severe income losses. It is more difficult to predict or determine the costs of these factors than the direct costs. Yet, they may well equal or even exceed the direct costs of bridge failure.

The estimated total cost of the Silver Bridge collapse at Point Pleasant, West Virginia, in 1967 was \$175 million (13). Considering the recent growth in litigation and general inflation, it would not be presumptuous to assume that today a similar failure would cost 10 times as much.

Bridge structural dilapidation (real or impending) also may impose great strains on the budget and work load of a bridge authority. In 1975, cracks were detected in the tie chords of the I-24 bridge over the Ohio River at Paducah. The bridge subsequently was closed for a year while butt welds in the tie chords were spliced (Figure 9). Traffic was rerouted over an old truss bridge that had been in service for 50 years. Due to the narrowness of that structure, many vehicles sustained major damage by sideswiping the truss beams. The cost of reworking the bridge

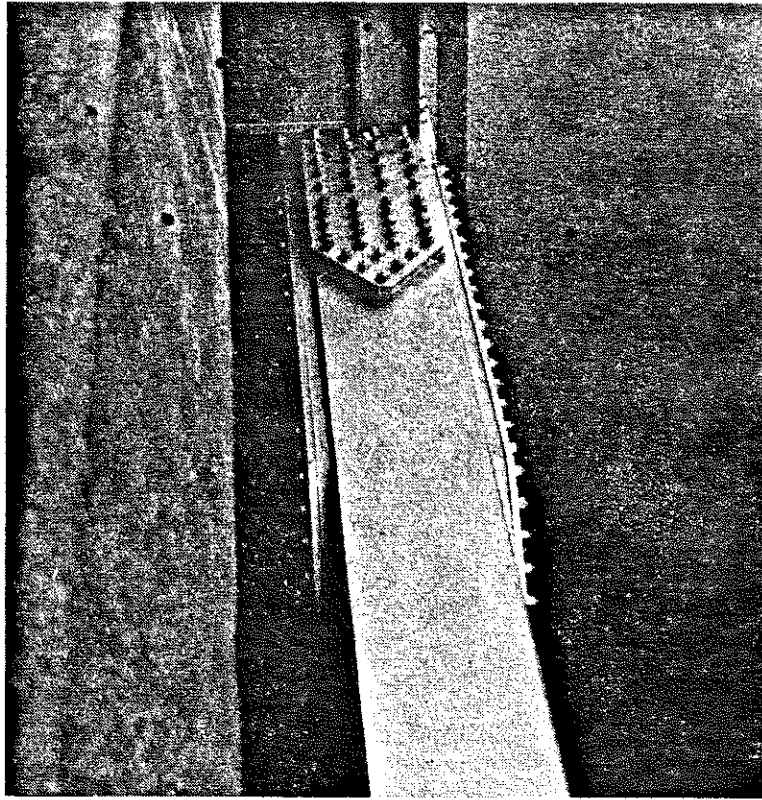


Figure 9. Splice over Butt Welds on the I-24 Bridge.

Table 1 — Risk of Fatality by Various Causes.

Type of Event	Individual Risk (Fatalities x 10^{10} /Exposure (Hour))
Flying, General Aviation	300,000
Brittle Failure of PP-Type Highly Stressed Bridge (40th to 70th Year, Given Survival After 40 Years)	35,000
Driving (All Accidents)	10,000
Brittle Failure of PP-Type, Highly Stressed Bridge (First 40 Years of Life)	8,000
Driving (Accidents Caused by Defective Motor Vehicle)	530
Brittle Failure of Moderate Stress Bridge (Worst-Case Estimate)	50*
Nuclear Power Plants	10*
Brittle Failure of Moderate Stress Bridge (Best Estimate)	2.2*
Natural Disasters	1

*These values are calculated from risk analyses and are not based on actual fatalities.
(Ref 18, pp. 15)

was \$3 million. The cost to motorists in gas consumption, time delays, and vehicle damage was never determined.

The level of funding for statewide routine periodic NDE surveillance may be approximated by:

$$\text{Level of NDE Funding} = \text{Risk (probability of failure)} \times \text{Consequences (cost of failure)}.$$

For example, if the statewide failure risk is 1 in a 1,000 per year and the anticipated maximum cost of failure is \$500 million, a justifiable funding level would be \$500,000. This is a gross simplification, but it demonstrates that appropriate funding levels may be deduced.

In most cases, linear elastic fracture mechanics (LEFM) is not a viable tool for use in predicting the maximum crack size that a bridge member will tolerate. This is due to the relatively low yield strengths of steels employed in bridges. However, during a significant portion of the growth of a NDE-detectable subcritical fatigue crack, a LEFM crack growth law is valid (Figure 10)(14). This relationship has the form:

$$da/dN = C(\Delta K)^n$$

in which

da/dN = fatigue crack growth rate per cycle,

ΔK = Stress-intensity range,

$$= K_{\max} - K_{\min},$$

C and n = material and test-related constants.

Knowing the cyclic loading rate and the initial crack size, the time required to achieve the critical crack size for failure may be determined. By selecting an appropriately sensitive NDE test method and by using the stress-intensity related fatigue-crack growth law, the frequency of NDE surveillance for a bridge may be determined. The interaction between the sensitivity of the NDE surveillance method and test frequency should be such that follow-up inspections will detect any growing fatigue cracks that were previously too small to be discovered before those cracks could cause structural failure (Figure 11).

Critical-sized cracks are defined as those that may cause failure in a structural member. It is difficult to specify reliable and universally recognized critical crack sizes for bridge steels subject to service stresses. Since an upper limit of crack size is necessary to insure the proper inspection interval, an approximation must be used. The critical flaw size might be set arbitrarily at some percentage of the material thickness. Therefore, the calculated test interval would be sufficiently short to (hopefully) allow detection of growing cracks before they transverse the plate thickness.

Figure 12 shows a surface-breaking crack on a fracture-critical bridge member made of ASTM A 514 steel. The crack, which did not penetrate the plate, was about three inches long. However, the live loading of the bridge member was extremely low.

Figure 13 shows the material cost and nondestructive inspection sensitivity for a turbine blade. What the graph reveals of relevance to bridges is that, below a certain inspection sensitivity, the costs are unjustified. When the maximum critical flaw size has been determined,

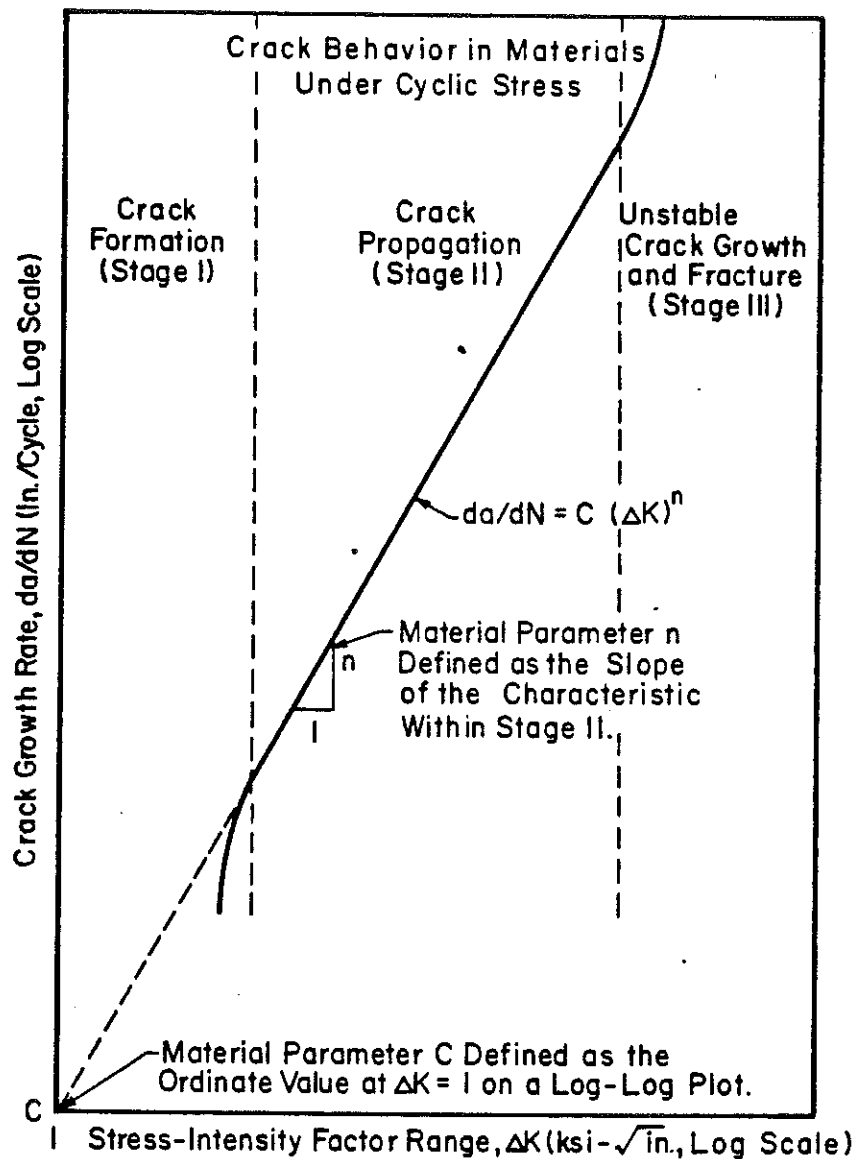


Figure 10. The Paris Fatigue Crack Growth Law (Ref. 13, pp 101).

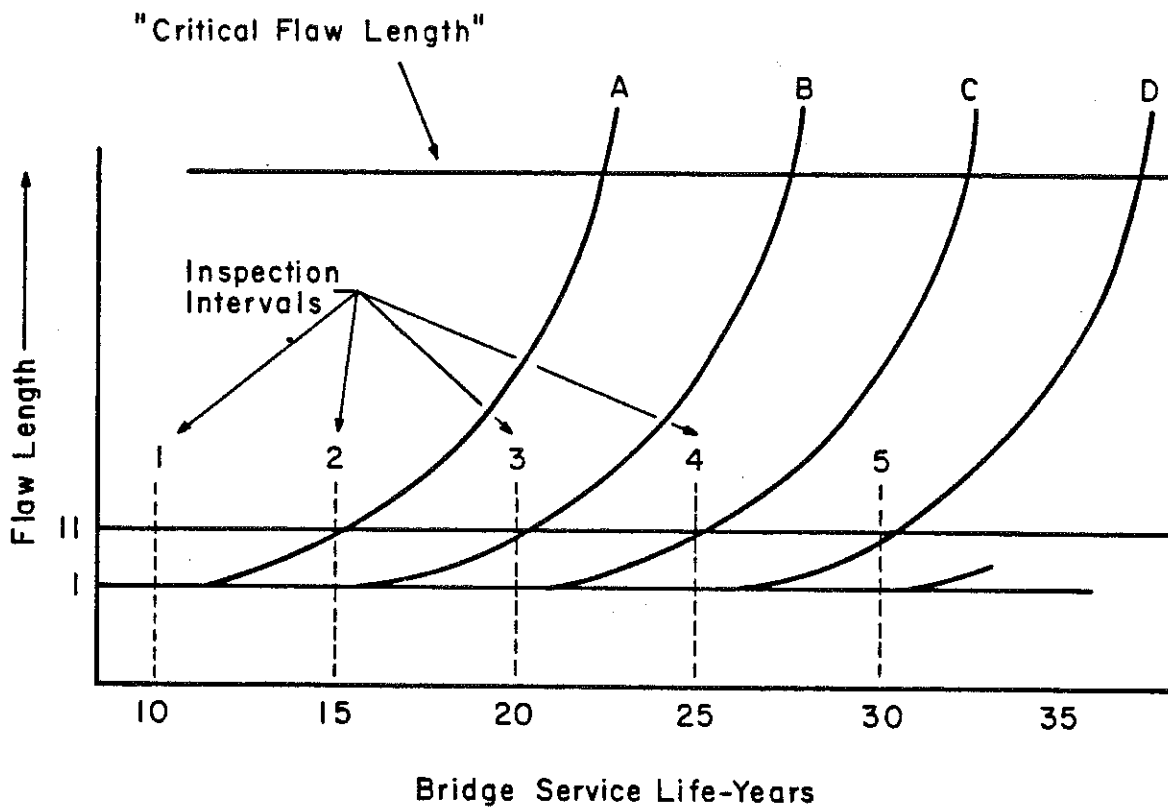


Figure 11. Life Extension Curves - Note at Higher Inspection Sensitivity Level (Flow Length I), the Inspection Interval Can Be Increased Compared To a Less Sensitive Inspection Level (Flow Length II) (Ref. 14, pp 19)

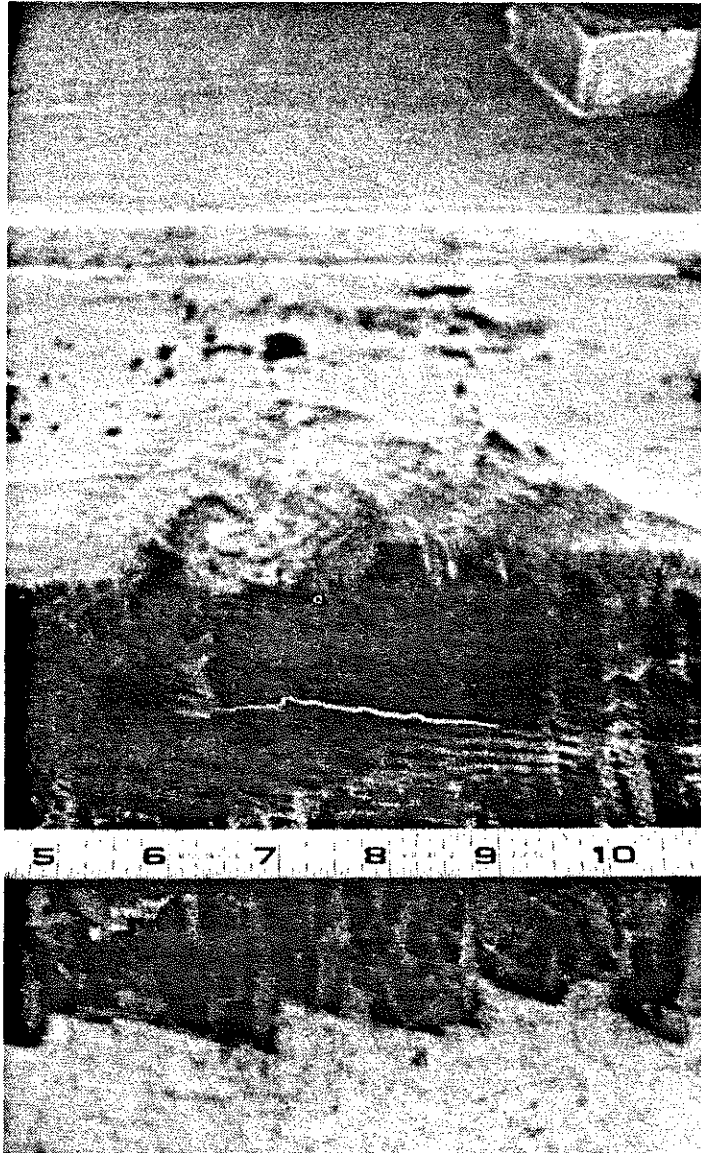


Figure 12. Surface Breaking Crack in a Bridge Member.

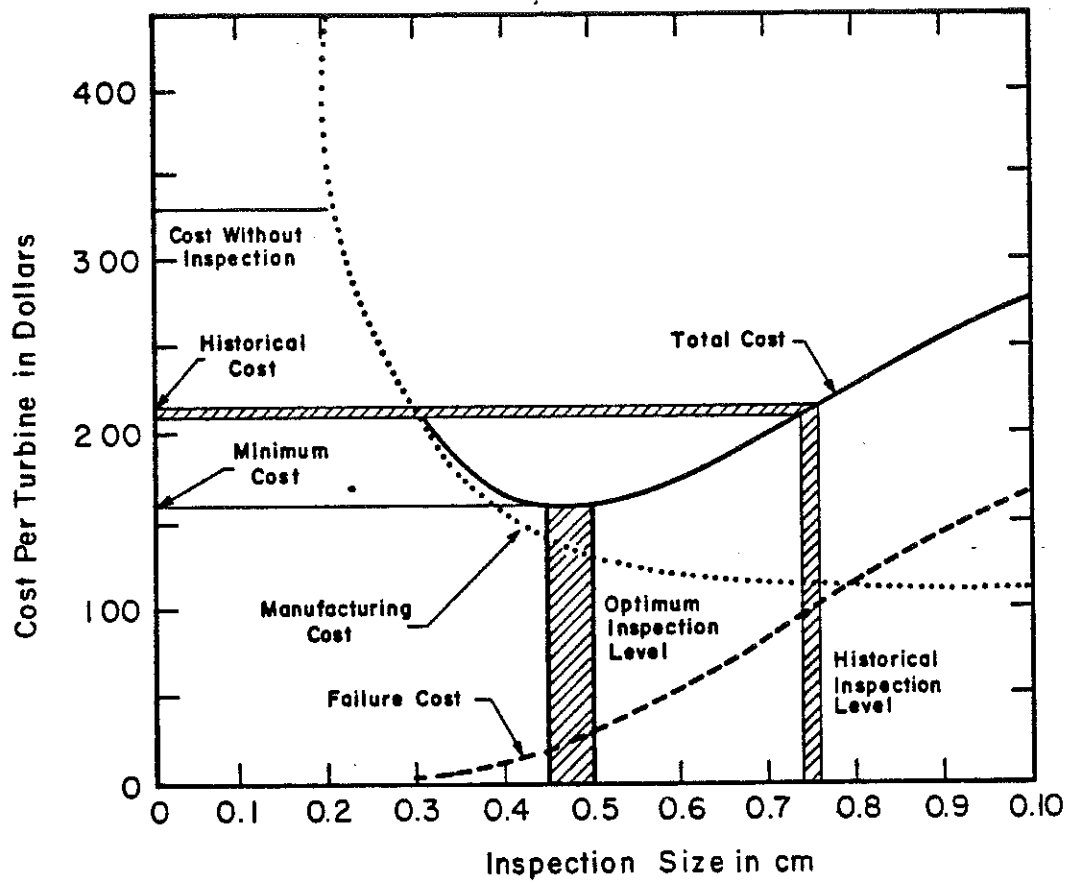


Figure 13. Costs Per Turbine Blade as a Function of Inspection Size.

it becomes obvious that detection of smaller defects will allow longer intervals between inspections. Unfortunately, any savings accrued by increasing the inspection interval may be more than offset by the increased cost of inspection and by possible problems with ambiguous test results.

It is likely that most routine, periodic nondestructive surveillance inspections could focus on surface-breaking cracks of lengths of ranging from 1/2 to 2 inches. Efforts to detect subsurface defects oftentimes may prove too costly and unproductive using geometric-based NDE methods.

Inspection strategies may be used effectively to reduce the inspection inventory. In fact, one major objective in performing this task is to eliminate bridges or structural members on bridges where either the risk is minimal or the results of structural failure are not catastrophic. A routine NDE surveillance program would require the combined efforts of highway or bridge authority design and maintenance units to achieve this goal.

In most states, either the design or maintenance units maintain the state bridge inventory. This may be an imposing task. Kentucky, for example, has 7,000 bridges inventoried. Of those, approximately 1,000 are classified as steel bridges. One hundred and sixty of those bridges have non-redundant or fracture-critical load-carrying members (FCMs).

It is the function of the design unit to analyze new and existing bridges to determine whether the structures have FCMs and to identify those members. The design unit also should determine the anticipated combined loadings and live loadings of those members.

The maintenance unit should have the responsibility of planning or approving inspection strategies. Maintenance personnel should plan all actual inspections, perform or oversee the field work, collect and analyze inspection results, and maintain records.

Where specialized technical support is required, a research organization is useful.

NDE METHODS

The most important components in performing routine NDE surveillance of bridges are the test methods employed and the operators who use them. Much of the success or failure of NDE techniques presently employed rests on the knowledge and skill of the equipment operator (Table 2). In practicality, therefore, when discussing most test methods, the NDE test method-operator couple should be considered (15).

As briefly mentioned earlier, most comprehensive nondestructive inspections involve deliberate, tedious work on the part of the equipment operator over the entire test surface of a structure. Much of the time consumed in those inspections is spent evaluating flaws in accordance with some formal inspection document such as the American Welding Society Code. This has several disadvantages: 1) test results may not be more significant than when simpler techniques were employed, 2) harmless flaws may be classified as rejectable defects, 3) the test method may induce operator errors (false-calls), 4) the test code may require extensive test site surface conditioning, and 5) the ability of the NDE test method to find the smallest reliably detectable defect size may be minimized. The net result of these factors is that field NDE

Table 2 – Factors Effecting NDI Proficiency.

Human	Physical
– Dexterity	– Environment
– Formal Training	– Inspection Rate
– Cognition	– Type of Structure
– Psychomotor Skill	– NDI Method
– Rational Ability	– Flaw Size & Density
– Motivation	– Part Geometry

work incorporating fabrication codes may be expensive and time consuming. Also, initial expectations about the correctness and usefulness of data derived from such nondestructive inspections may prove to be so discouraging as to curtail plans for other nondestructive inspections.

When routine, periodic NDE surveillance of bridges is attempted, the bulk of the inspection effort must be placed on scanning or searching for defects. Productivity becomes a more important consideration and some trade-off must be made between inspection rate and test sensitivity. While this may result in shorter inspection intervals for each bridge compared to code-based flaw-evaluation inspections, it is more than offset by the greatly reduced cost per inspection. Another advantage is that the scanning operation may be "tailored" to a known minimum defect size and that indications from smaller nonrelevant flaws can be neglected. The test rationale is established from the previously discussed fatigue-crack growth calculations and from NDE qualification testing of flawed specimens using the NDE procedure, equipment, and operators to be employed in the actual field tests. In the testing of large bridges, where thousands of linear feet of welds need to be inspected, this approach will yield the maximum benefit. The NDE scanning method employed may provide more useful information concerning the physical dimensions of existing defects than a code-based flaw-evaluation technique. Also, the scanning method may allow inspection of the bridge with minimum surface conditioning of test areas.

In either scanning or flaw-evaluation NDE tests of in-service bridges, several test-method attributes are desirable. Test results should be easy to document, with direct hard-copy output being most beneficial. Test results should be confirmable by use of another NDE method. It is desirable to be able to confirm all rejectable indications and to perform validation tests on inspected "defect-free" areas using another NDE method. The test method should not require time-consuming surface conditioning of test areas. Paint removal, cleaning, and grinding may be almost as time consuming and expensive as the NDE work. Also, those surfaces must be repainted following the inspection, adding another expense. The NDE test equipment should be portable and should allow the operator sufficient time to inspect large remote areas before having to return to his base of operations for resupply or recalibration.

There are three general types of nondestructive inspections applicable to bridges: 1) surface indication methods, 2) subsurface indication methods, and 3) acoustic emission methods. The first two entail geometric defect sizing. The latter detects subcritical flaw activity.

Relevant surface methods include dye penetrants (visible and fluorescent), magnetic particle (visible and fluorescent), and eddy current. In many cases, these methods may be used effectively in locations where surface-breaking cracks are sought.

The first two methods require nominal capital equipment outlay and may not necessitate extensive formal operator training. Unfortunately, those methods require paint removal and cleaning to be effective, which in turn increase inspection costs. Also, the consumption of expendable supplies, penetrants and ferrous powders, may prove expensive if a large number of bridges is inspected.

Visible surface NDE tests are effective in direct sunlight. However, in heavily shaded areas (under a bridge deck) or closed areas (inside a box beam), supplemental lighting is necessary (Figure 14). At those locations, fluorescent inspection may prove more beneficial. Fluorescent testing cannot be performed effectively under direct sunlight. On at least one occasion, a highway authority has performed fluorescent magnetic-particle testing on the chords of a large arch bridge by inspecting the structure at night.

Eddy-current testing may prove more beneficial for surface-crack inspection than either the dye-penetrant or the magnetic-particle methods. Eddy-current testing requires minimal surface conditioning of test areas. Portable eddy-current devices are expensive. However, they do not require significant expenditures for consumable supplies. Also, the units allow operators to work on remote portions of bridges for extended periods. Some operator training is required, but this training does not need to be as extensive as that for ultrasonic operators using code-based flaw-evaluation techniques.

Several eddy-current or magnetic-field disturbance units have potential for inspecting welds. A typical portable commercial unit uses a CRT screen to differentiate between the presence of cracks and the lift-off effects of irregular weld surfaces. The Federal Highway Administration has sponsored development of the Magnetic Crack Definer by the Southwest Research Institute, San Antonio, Texas, to locate and measure surface cracks. The unit is designed to be used by relatively inexperienced inspection personnel and, therefore, has simplified controls and readouts.

The two main subsurface methods, radiography and ultrasound, also use geometric defect sizing. Transmission radiography has not been considered for routine NDE surveillance due to its high cost, low productivity, and safety requirements.

Real-time radiography shows promise for inspecting wrapped strands on cable-stayed bridges. This type of radiography can penetrate superficial wrapping and permit observation of the profile of the wires. As shown in Figure 15, the strand on the left side of the photograph has a rough profile due to corrosion of the zinc coating. The profile of the uncorroded wire, shown to the right, is smooth except for a sawcut, which was deliberately made in this specimen.

Ultrasonic inspection is useful for both scanning and flaw-evaluation inspections. Generally, ultrasonic testing requires significant expenditures in equipment and personnel training. Due to its versatility, however, it should be considered an essential ingredient in any routine NDE surveillance program, if only to be used to confirm indications by other NDE methods or to measure the depth of surface-crack indications. The amount of surface conditioning required to inspect welds ultrasonically may vary between tests. Scanning inspections may require little surface preparation compared to flaw-evaluation techniques, which may require paint removal.

In more recent bridges, fabricated to ultrasonic quality-assurance standards, ultrasonic techniques may prove useful in inspecting for relatively small subsurface defects. In older bridges, the presence of laminations may curtail its effectiveness by creating "false-calls" and slowing the inspection rate. This may be mitigated somewhat by using less-sensitive test procedures.

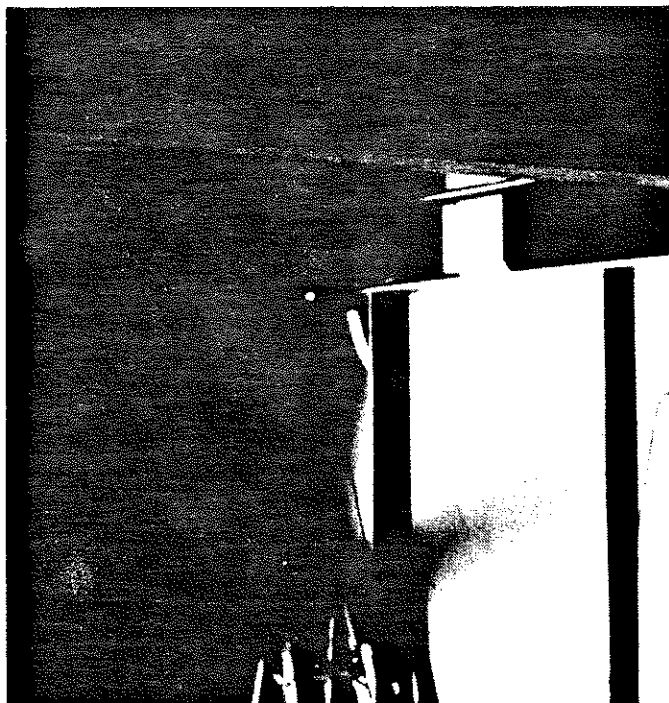
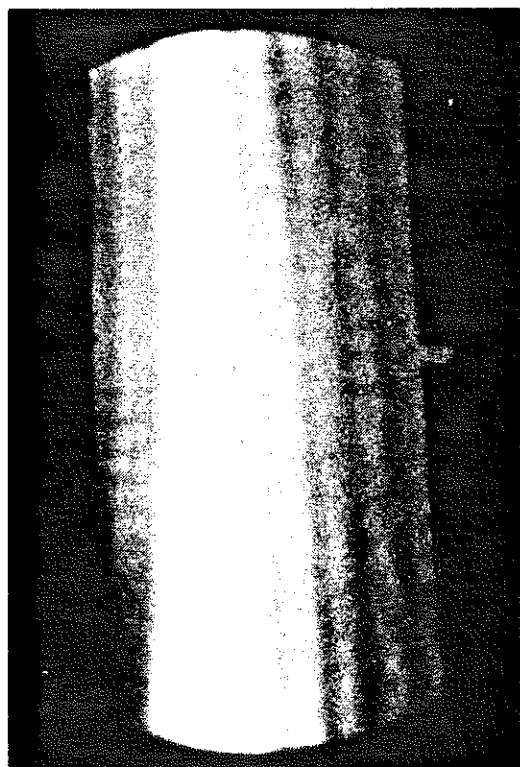


Figure 14. Shaded Area under a Bridge Deck.



**Figure 15. Real Time Radiograph of Both Corroded
and Uncorroded Galvanized Wire Strands.**

Presently, some questions exist as how to best use ultrasound for scanning. The distance-amplitude correction method is readily adaptable for scanning. Instead of evaluating each flaw, a "go/no-go" approach may be adopted using some minimum flaw size as the reference reflector. For scanning, the flaw size selected would be large enough to preclude the need for closely inspecting each ultrasonic indication. The probe-movement technique also may prove worthwhile, especially for mapping defects.

The Federal Highway Administration has sponsored development of the Acoustic Crack Detector by the Southwest Research Institute for subsurface crack detection on bridges. This device uses gated ultrasound to detect cracks. As with the Magnetic Crack Definer, the device is designed to minimize operator requirements.

Acoustic emission testing shows much promise as a tool for scanning bridges. Among its advantages are: 1) only active, growing defects will produce acoustic emissions; 2) the bulk of the physical work may be performed by relatively unskilled labor; 3) large areas of a bridge may be scanned simultaneously; 4) a very small defect may be detected, maximizing inspection intervals; 5) minimal surface conditioning on the structure is required; 6) while acoustic emission testing is in progress, inspection personnel may attend to other tasks (a "set and forget" feature); 7) active defects may be accurately located along the test surface; 8) the equipment may produce hard-copy records at the test site; and 9) the method lends itself well to the performance of low-cost, high-productivity nondestructive inspection, necessary traits for routine NDE surveillance.

Acoustic emission testing cannot be used to geometrically define defects. Any acoustic emission source must be located and sized using a conventional NDE method. Also, the test structure must be loaded sufficiently to assure crack growth or fretting. When normal traffic is used to drive cracks, extended monitoring periods may be required to detect crack-related acoustic emission activity. This is due to the fact that acoustically "quiet" periods are often encountered during early and intermediate stages of fatigue crack growth.

Noise sources have been a prohibitive problem for acoustic emission monitoring of bridges in the past (16). However, the Kentucky Transportation Research Program has been successful in testing bridges using an advanced acoustic-emission weld monitor developed for the FHWA by GARD Corp. of Niles, Illinois (Figure 16). This device uses micro-processors to locate and categorize acoustic-emission sources such as cracks.

CLOSURE

Over the next two years, the Kentucky Transportation Research Program will continue to field-evaluate the NDE equipment and test methods discussed in this paper. In addition to using those NDE methods to detect cracks on bridges, laboratory tests will be conducted using those methods to inspect preflawed specimens to determine their usefulness. Comprehensive inspection strategies will be formulated. Data will be obtained on practical inspection rates achieved with each NDE method. Also, related field experiences, both failures and

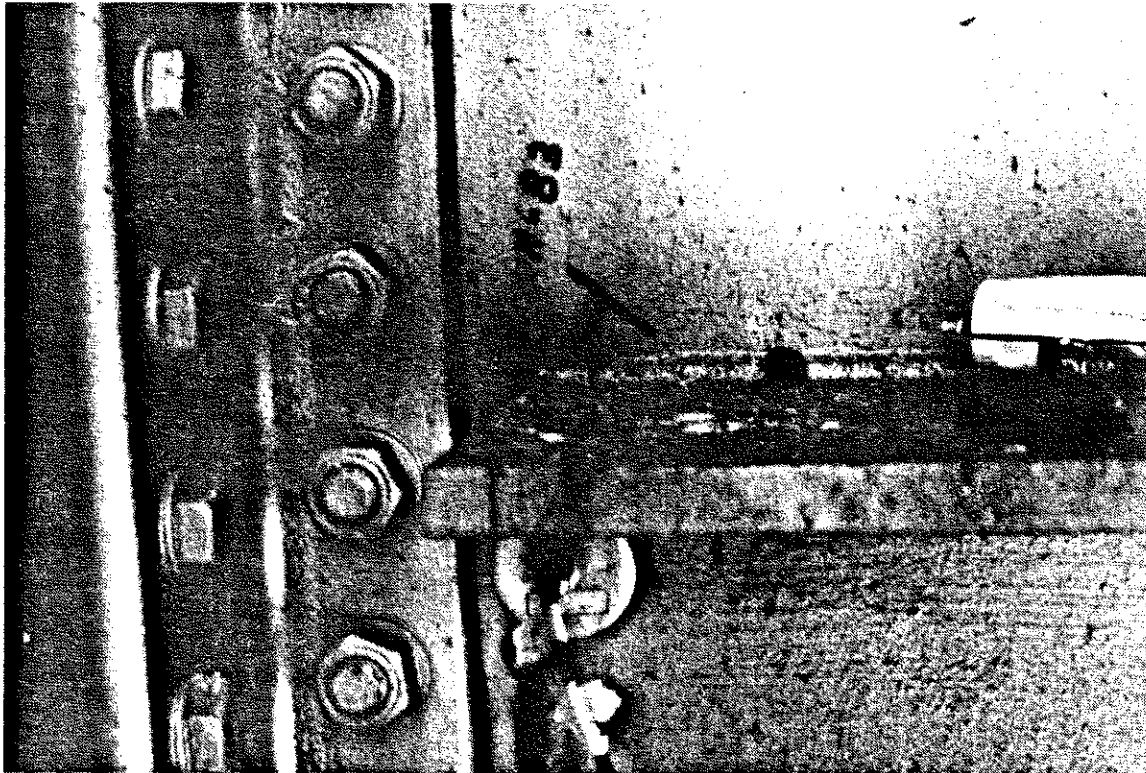


Figure 16. Acoustic Emission Monitoring of a Crack.

successes will be recorded and documented. When the study is completed, recommendations and guidelines will be prepared for performing routine NDE surveillance of bridges.

Ten years ago, the high cost of conducting periodic, routine NDE inspections of in-service bridges made such work almost unthinkable. However, NDE techniques and inspection procedures that will significantly reduce those costs and make such testing a reality are rapidly evolving. When those techniques and procedures are technically mature and proven, it would be desirable for all bridge authorities to perform such inspections.

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